CHAPTER 3

SPECIMEN DESIGN AND CONSTRUCTION

Two test specimens were constructed at the UC Berkeley Richmond Field Station and tested on the shake table at the UC Berkeley Earthquake Engineering Research Center. The specimens were designed to represent a reinforced-concrete (RC) flat plate system and a post-tensioned (PT) flat plate system, respectively, for use as a non-participating system in high seismic regions, or intermediate frames in moderate seismic regions, in both the U.S. and Japan. Each specimen consisted of a two-by-two bay slab-column frame, two stories high (**Fig. 3-1**). Due to dimensional limitations of the shake table, a scale factor of approximately one-third was used for both specimens. The specimens included six columns, with two full spans in the east-west (E-W) direction, and one full span and two half-spans in the north-south (N-S) direction (**Fig. 3-1**). This geometry was selected to provide symmetry and stability, to maintain typical span-to-depth ratios used for each type of construction (RC and PT), and to provide appropriate boundary conditions for the uniaxial shake table tests.

Design of the test specimens was aided by studies conducted on prototype specimens designed to satisfy typical requirements for construction used on the west coast of the US. The prototype buildings consisted of multi-story flat plate systems with either perimeter frames or core walls, or both (dual system), designed according to UBC-97 (1997) requirements for Zone 4 (high seismic region) (**Fig. 3-2**). The non-participating flat plate

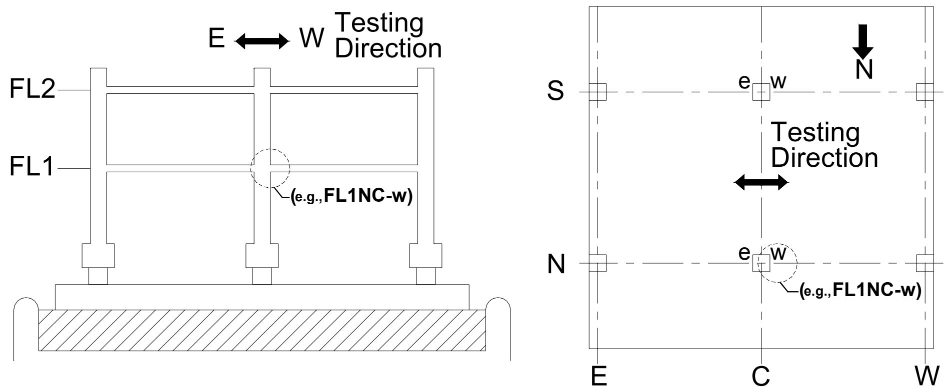


Fig. 3-1 Test specimen on shake table

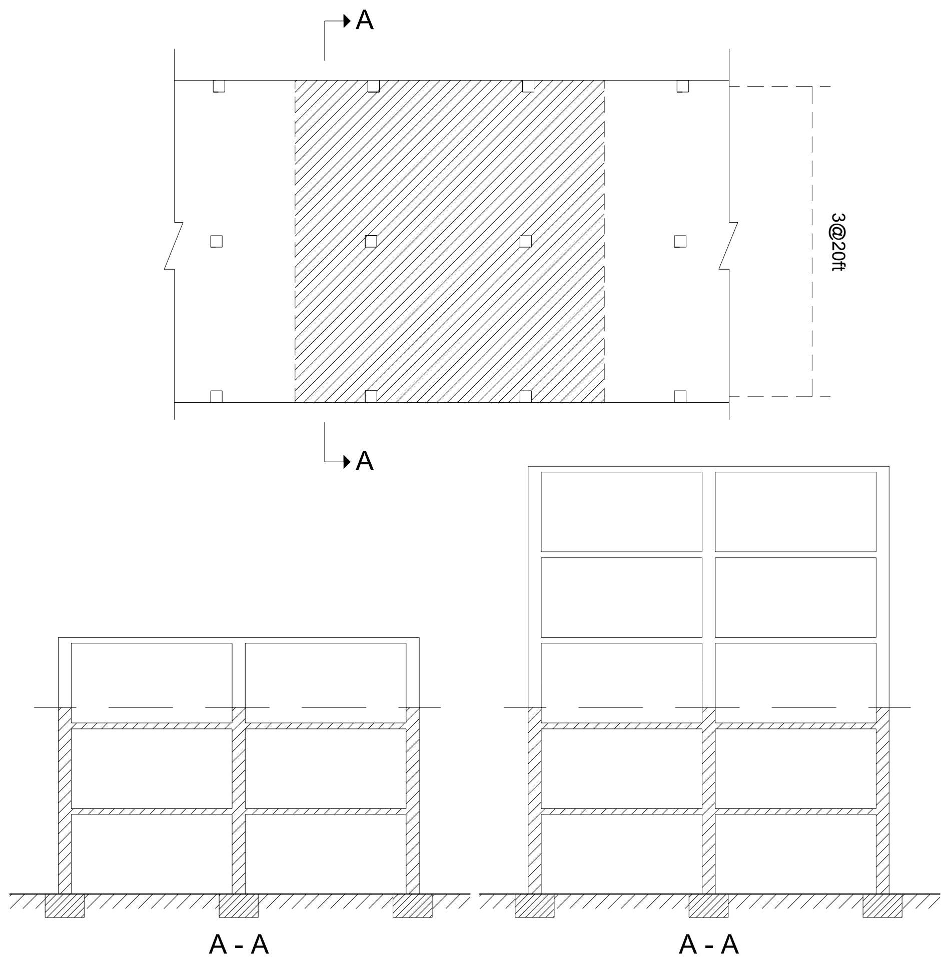


Fig. 3-2 Prototype buildings and shake table specimens (Hatched)

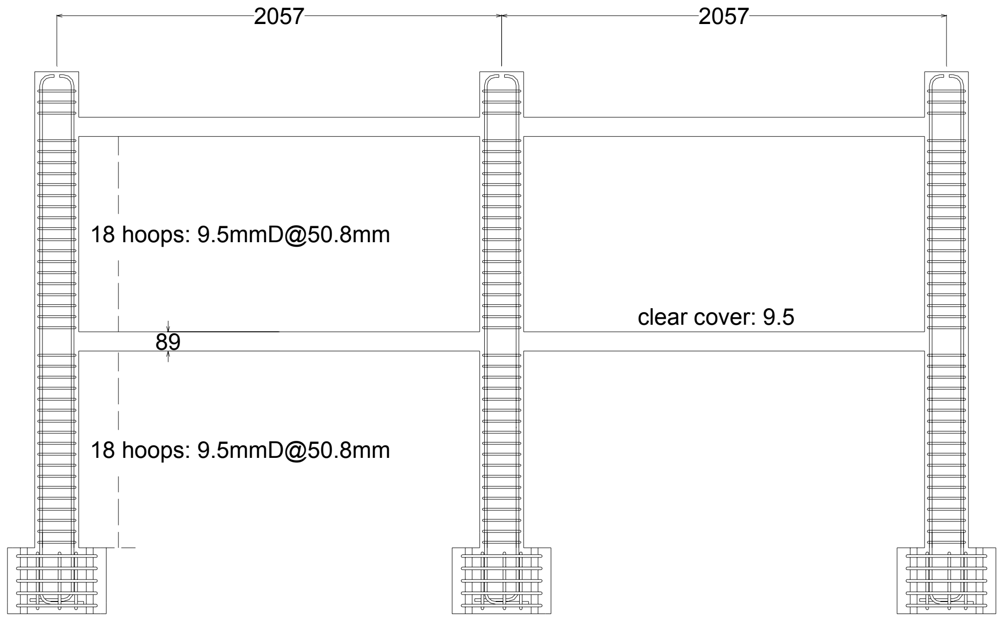
Fig. 3-3(a) RC specimen

Fig. 3-3(b) PT specimen

systems for Zone 4 are commonly used for gravity loads only. Slabs were designed to satisfy the requirements of ACI 318-02 requirements ("Building", 2002), as well as to meet specific objectives of the project, that is, to investigate the use of slab shear reinforcement at the slab-column connection regions, as described in detail in the following subsections. In general, column flexural strengths were selected to exceed the sum of the nominal moments for the slabs framing into a column, so that yielding and damage would be concentrated in the slab-column connection region. In addition, the connection region was designed such that yielding of slab reinforcement was anticipated, followed by punching failure of the connection region. Each of the specimens (**Fig. 3-3**) is described in detail in the following subsections with more detailed calculation is provided in **Appendix A**.

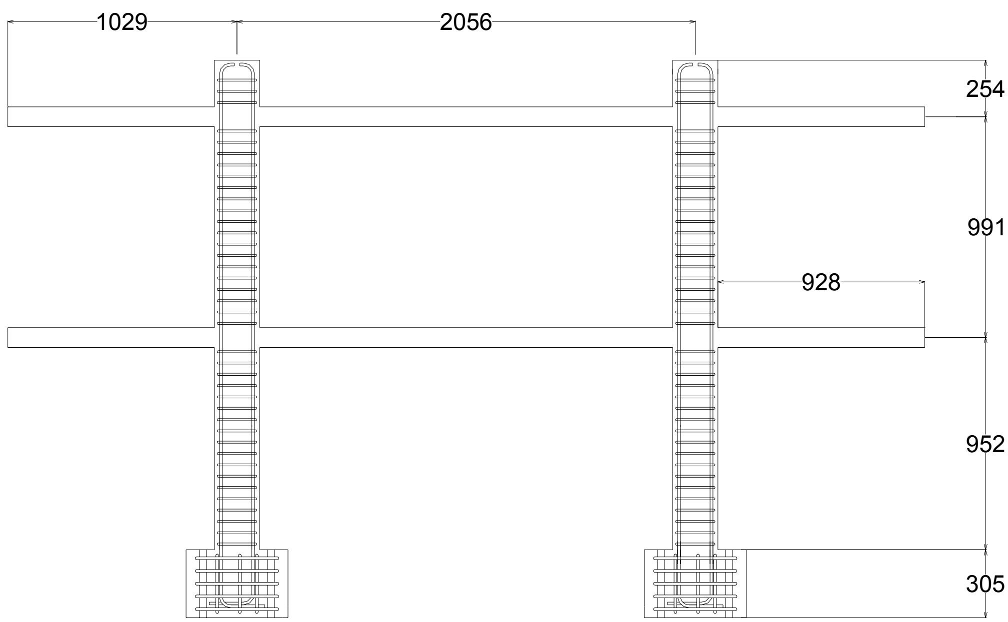
3.1 RC Specimen

Typical slab span-to-depth ratios for RC flat plate construction are roughly 20 to 30 (25-30 in the U.S., 20-25 in Japan); therefore, a bay width of 2.06 m (6 ft. 9 in.) and a slab thickness of 89 mm (3.5 in.) were selected, resulting in a span-to-depth ratio of 23.1 for the test specimen. Elevation views of RC specimen are shown in **Fig. 3-4**. A column cross section of 203 mm × 203 mm (8 in. x 8 in.) was selected to produce a column depth-to-span length ratio (c_1/l_1) of 0.1, which is representative of typical values for RC flat plate systems. Gravity load consisted of the slab self-weight (2.2 kPa; 45 psf) and lead weights affixed to the slab to provide an additional 5.7 kPa (120 psf) to represent the



FRAME N, FRAME S (See Fig. 3-1)

Fig. 3-4(a) Elevation view of RC specimen (Frame N, S)



FRAME W, FRAME C, FRAME E (See Fig. 3-1)

Fig. 3-4(b) Elevation view of RC specimen (Frame W, C, E)

self-weight of the prototype floor slab of 267 mm thickness (6.5 kPa; 135 psf), as well as an additional gravity load of 1.4 kPa (30 psf). The resulting gravity shear ratios on the slab critical section adjacent to the columns for the interior and exterior connections were $0.25V_c$ and $0.20V_c$, respectively, which are fairly representative typical of reinforced concrete flat plate construction (0.25-0.40 in the U.S., 0.15-0.30 in Japan).

Column flexural strengths for the RC specimen were selected to ensure that yielding and damage would be concentrated in the slab-column connection region versus within the column. Resulting column cross sections were 203 x 203 mm (8 in. x 8 in.) reinforced with 8 − # 4 (d_b = 12.7 mm; 0.5 in.) longitudinal bars, with a nominal yield stress of 414 MPa (60 ksi) (**Fig. 3-5**). The column nominal moment was 34 kN-m (300 in.-kips) for a zero axial force. For the second floor (roof) level, the column nominal moment was approximately equal to the probable negative moment ($M_{pr} = (6/5)M_n$) associated with a slab yield line (**Fig. 2.3**) for an exterior connection (33 kN-m; 290 in.-kips) and an interior connection (where the sum of the probable negative, 22 kN-m, and positive, 14 kN-m, is 36 kN-m for the column strip). Column moment capacities at the first floor level were sufficient to resist probable moments for slab yield lines across the full slab width, as the column extended both above and below the slab. All columns were provided with #3 (d_b = 9.5 mm; 0.375 in.) closed-hoops spaced at 50.8 mm (2 in.) on center (corresponding to the maximum spacing of 152 mm or 6 in. allowed for the prototype column) per ACI 318-02 21.4 (“Building”, 2002). The column hoops were selected to

provide sufficient shear strength to resist the column shear developed for a sway mechanism (**Fig. 2-3**). No column splices were used.

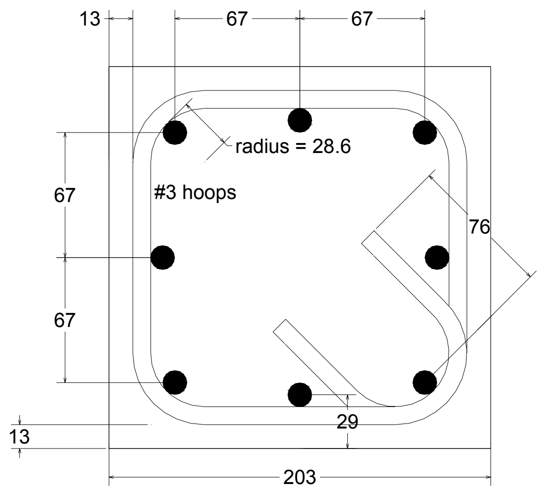


Fig. 3-5(a) Details of column

Fig. 3-5(b) Column reinforcement

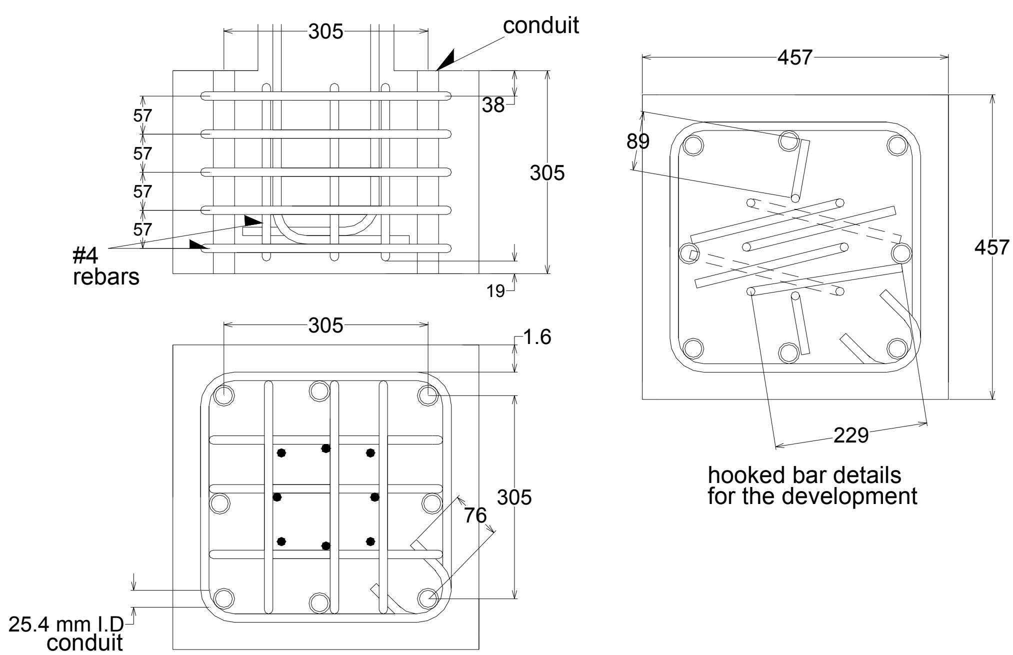


Fig. 3-6(a) Details of footing

Fig. 3-6(b) Footing reinforcement

To monitor base shear and overturning moment, and the unbalanced moment transferred at each of the first-story slab-column connections during testing, individual footings supported on load cells were used for each column. Eight, 25.4 mm inside diameter (one-inch I.D.) conduits were provided in each column footing to allow for attachment of the specimen to the tri-axial load-cells as shown in **Fig. 3-6**. The depth of the footings was selected to exceed the development length l_{dh} of the column longitudinal bars. Footings were well-confined, with 5 − # 4 (d_b = 12.7 mm; 0.5 in.) horizontal and 6 − # 4 (d_b = 12.7mm; 0.5 in.) vertical stirrups provided (**Fig. 3-6**).

Slab flexural reinforcement was initially designed to resist the slab moments due to gravity load determined using the direct design method as described in ACI 318-02 13.6 ("Building", 2002). However, given the specimen proportions (i.e., span-to-depth ratio, gravity shear ratio, etc.), punching failure was not expected to occur. Given the objectives

of the project, additional slab flexural reinforcement was provided to increase the unbalanced moment demand such that the use of slab shear reinforcement at the slab-column connection was required (**Fig. 3-7**). As a result, the unbalanced moment demand determined from the negative or positive yield moments (for exterior connections; **Fig. 3-8(a)** and **3-8(b)**) within the column strip, or the sum (for interior connections; **Fig. 3-8(c)**), were greater than the nominal unbalanced moment capacity ($M_{n,unb}$) that would cause punching failure at the slab-column connection in the absence of shear reinforcement. With the addition of slab shear reinforcement, yielding of slab reinforcement within the column strip was anticipated (**Table 3-1**); however, punching failures were expected to occur prior to yielding of slab reinforcement across the full width of the slab. The capacity of the connections to transfer unbalanced moment were based on the eccentric shear stress model of ACI 318-02 ("Building", 2002), that is, $M_{n,unb}$ is determined using the following relation:

$$v_n \equiv \frac{V_g}{b_o d} \pm \frac{\gamma_v M_{n,unb} c}{J_c} \tag{1}$$

where v_n (= $1/3\sqrt{f'_c}$) is the nominal shear capacity at the connection in the absence of shear reinforcement, V_g is the gravity force to be transferred from the slab to the column, b_o is the perimeter of critical section, d is the effective slab depth, γ_v is the fraction of unbalanced moment transferred by eccentric shear, c is the distance from the centroid of the critical section to the perimeter of the critical section that results in the smallest value of $M_{n,unb}$, and J_c is the polar moment of inertia of the slab critical section.

Table 3-1 Shear demand and capacity for column and outer critical sections

	RC – Int. (γ_v = 0.25)	RC – Int. (γ_v = 0.40)	RC – Ext. (γ_v = 0)	RC – Ext. (γ_v= 0.384$^{\Diamond}$)	PT – Int. (γ_v = 0.40)	PT- Ext. (γ_v= 0.386$^{\Diamond}$)
$v_{u,direct}$	0.434	0.434	0.345	0.345	0.641	0.555
$v_u^{1\,*}$	1.710/-0.848	2.475/-1.613	0.738/0.041	2.772/-3.468	2.613/-1.331	3.509/-2.972
$v_u^{2\,*}$	·	0.614/-0.234	·	0.855/-0.317	0.889/-0.221	1.103/-0.531
$v_u^{1\,**}$	2.392/-1.531	3.571/-2.703	1.007/-0.131	4.096/-5.288	2.896/-1.613	3.896/-3.358
$v_u^{2\,**}$	·	0.841/-0.469	·	1.207/-0.600	0.972/-0.303	1.200/-0.614
ϕv_c^1	1.482	1.482	1.482	1.482	1.648	1.648
ϕv_c^2	·	0.745	·	0.745	1.110†	1.110†
ϕv_n^1	2.227	2.227	2.227	2.227	2.965†	2.965†

All units – [kPa].
$^{\Diamond}$ γ_v = 0.305 (RC), 0.313 (PT) for the outer critical section.
^{1} on critical section at *d/2* from column face.
^{2} on critical section at *d/2* from outermost peripheral line of studs.
* Based on $M_{u,unb} = M^{\mp}_{y,cs}$ (+ $M^{\pm}_{y,cs}$) ; (sum of yield moments within the column strip).
** Based on $M_{u,unb} = M^{\mp}_{y,fs}$ (+ $M^{\pm}_{y,fs}$) ; (sum of yield moments across the full span).
† Upper limit (ACI 421.1R-99).

To resist the fraction of the unbalanced moment transferred by flexure $\gamma_f M_{u,unb}$, as well as satisfy the requirements in ACI 318-02 ("Building", 2002) 21.12.6.3, about 60% of the slab reinforcement in the column strip was placed within c_2+3h for both interior or exterior connections (**Fig. 3-9(a)** and **3-9(b)**), where c_2 is the column dimension perpendicular to the direction of the applied loads and h is the slab thickness. The top and bottom slab reinforcement ratios within c_2+3h of 1.38% and 0.78%, respectively, did not exceed $0.75\rho_b$ = 2.14%, where ρ_b is the balanced steel ratio. In addition, ACI 318-02 requirements of Section 21.12.6 were satisfied, as 71% of top bars were placed within the column strip and all bottom bars were continuous (**Fig. 3-9(c)**). Two continuous bottom bars were placed within the column cage in accordance with ACI 318-02 13.3.8.5.

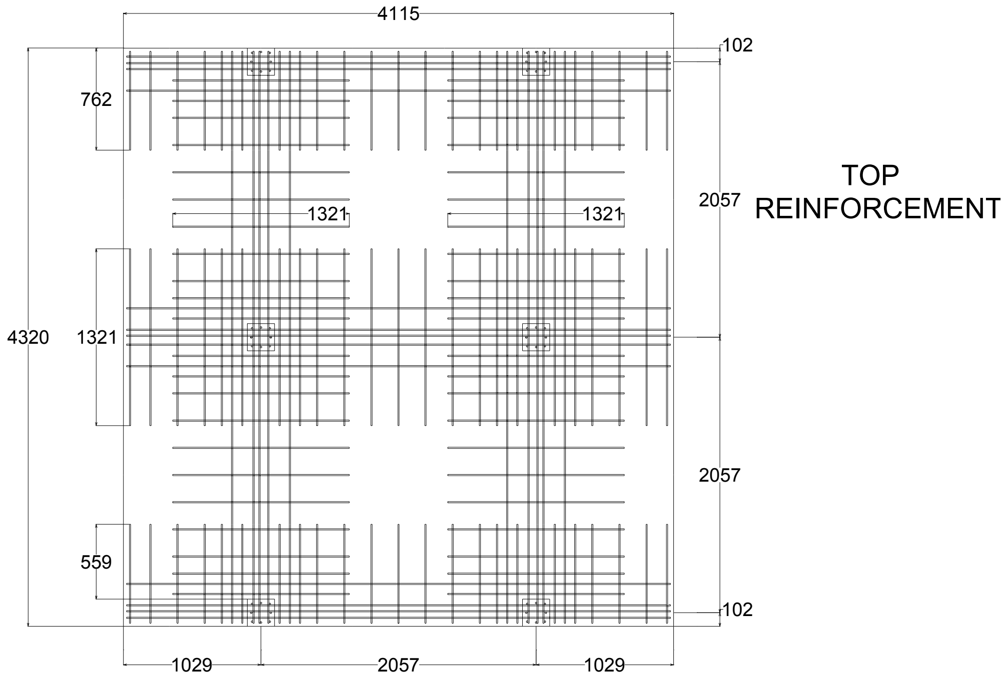


Fig. 3-7(a) Top slab reinforcement of RC specimen

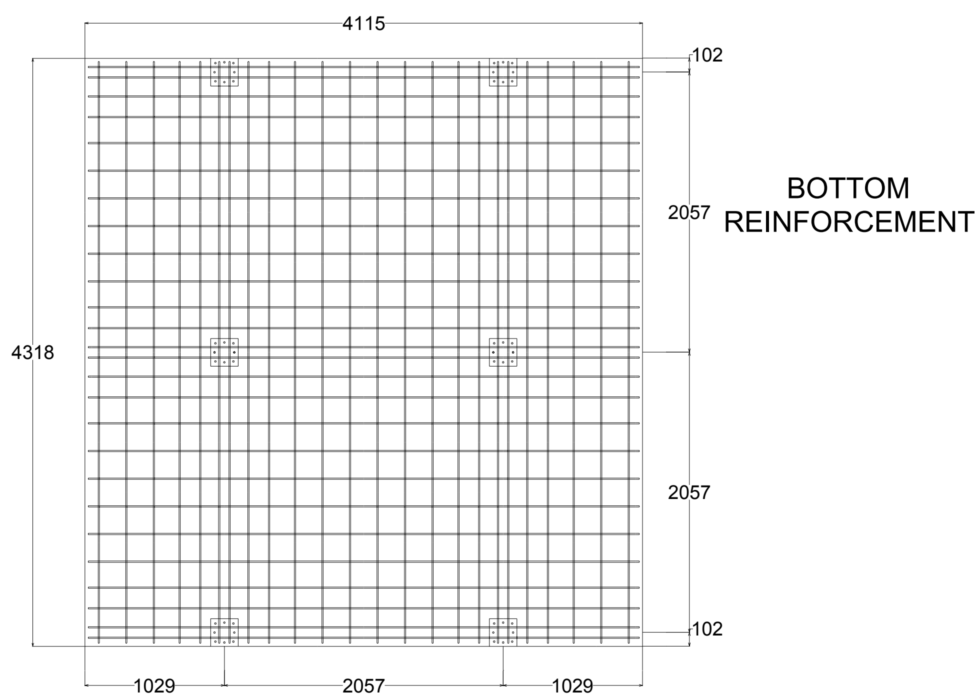


Fig. 3-7(b) Bottom slab reinforcement of RC specimen

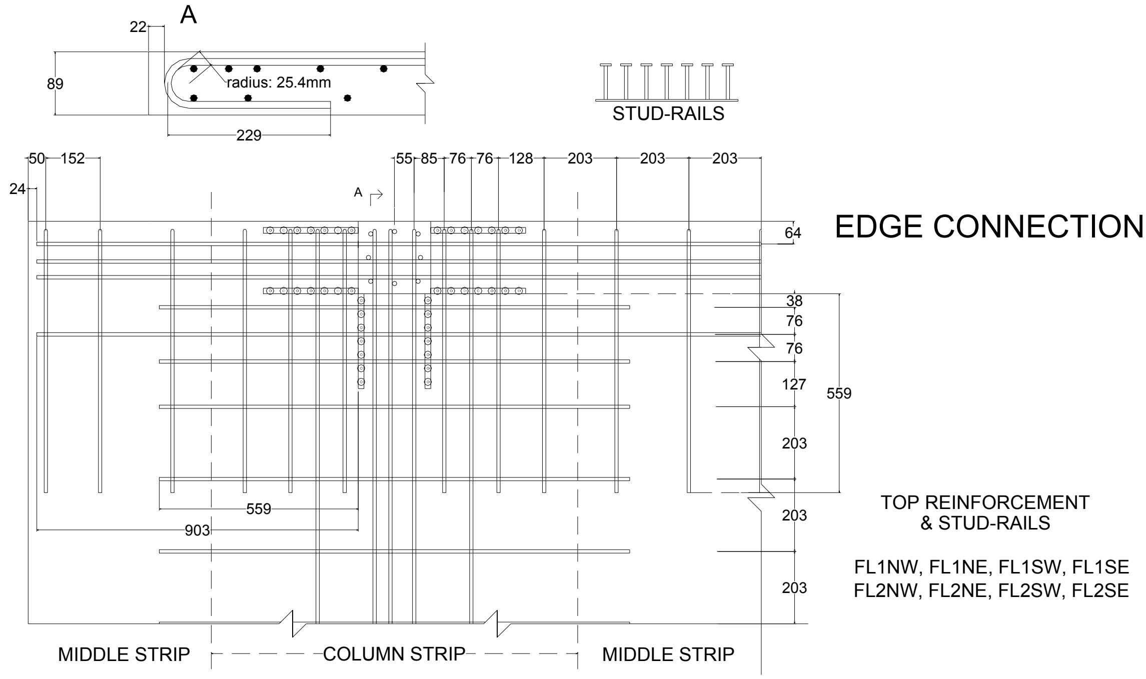


Fig. 3-8(a) Details of top reinforcement (RC specimen, exterior connection)

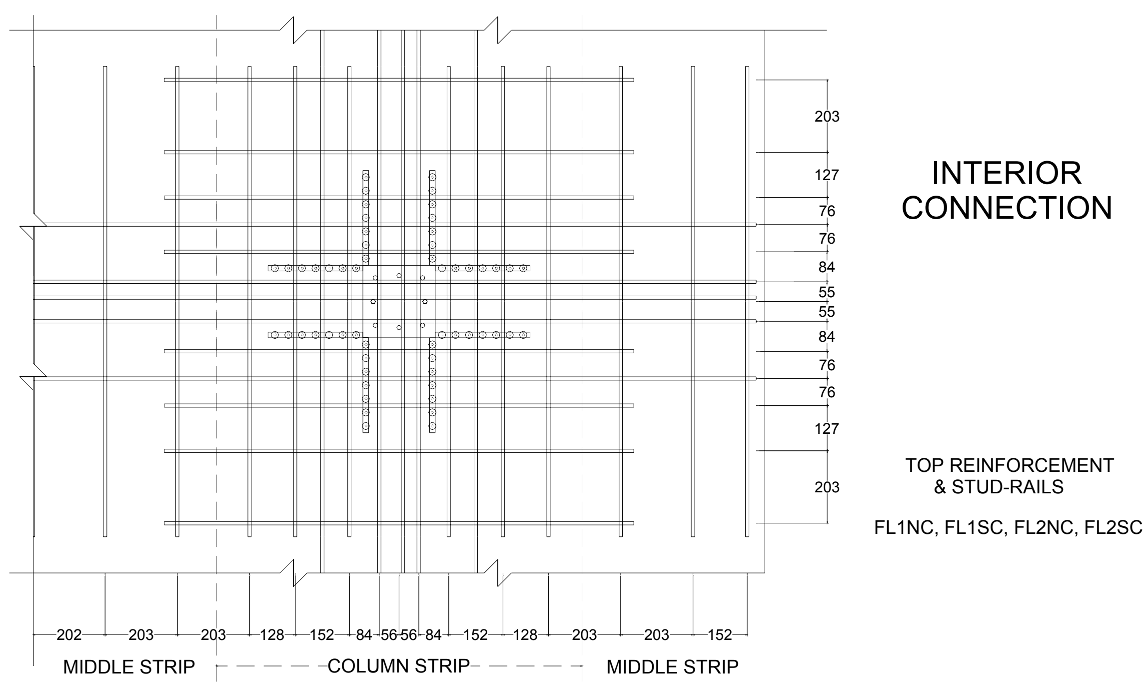


Fig. 3-8(b) Details of top reinforcement (RC specimen, interior connection)

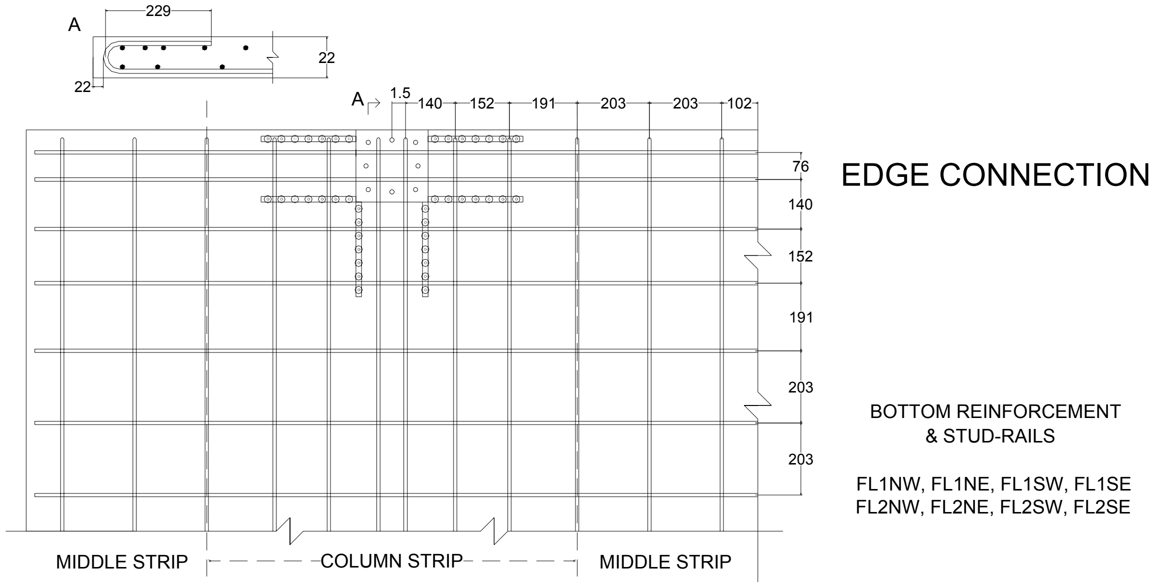


Fig. 3-8(c) Details of bottom reinforcement (RC specimen)

The ratio of top to bottom reinforcement for unbalanced moment transfer within the transfer width of c_2+3h was selected as approximately two as recommended in ACI 318-02 R.13.5.3.3 (**Fig. 3-8(b)** and **3-8(c)**), and slab minimum reinforcement was provided to satisfy ACI 318-02 7.12.2.1. The slab reinforcement consisted of # 3 (d_b = 9.5 mm; 0.375 in.) deformed bars with a yield stress of 458 MPa (66.4 ksi) having 180-degree standard hooks at the slab edge (**Fig. 3-8(a)**). Minimum concrete clear cover was 9.5 mm ($1.0d_b$) for both top and bottom #3 slab reinforcement. The slab reinforcement provided is summarized in **Table 3-2**.

Shear reinforcement, in the form of stud-rails, was selected because available test results have shown them to be effective (Ghali and Hammill, 1992; Megally, 1998) and they are commonly used. Design shear strength of the shear reinforced connections was based on

Table 3-2 Reinforcing steel ratios for various conditions

	Column Width	c_2+3h	Column Strip	Middle Strip	Full Span
Width (b)	203 mm	470 mm	1,029 mm	1,029 mm	2,057 mm
Top Bars	1.38	1.38	1.00	0.45	0.72
Bottom Bars	0.92	0.79	0.63	0.45	0.54

All – [%], $\rho = A_s/(bh)$, h = 76.2 mm

Fig. 3-9(a) Exterior connection (RC)

Fig. 3-9(b) Interior connection (RC)

Fig. 3-9(c) Slab reinforcement of RC specimen

ACI 318-02 ("Building", 2002) and ACI 421.1R (1999) requirements. Stud-rails were provided to resist combined direct gravity shear stress $v_{u,direct}$ and eccentric shear stress $v_{u,unb}$ due to the unbalanced moment transferred by eccentric shear on the slab critical section located at *d/2* from the column face (column critical section in **Fig. 3-10**). Outside the shear-reinforced region (outer critical section in **Fig. 3-10**), shear resistance is provided by concrete alone. Given the slab reinforcement for flexural and shear, and the column dimensions and slab thickness, the shear capacity at the column critical section is sufficient to resist the slab positive and negative yield moments on opposite sides of an interior column for the reinforcement placed within the column strip; however, punching failure was expected to occur prior to the development of a flexural yield lines across the full width of the slab (**Table 3-1**).

Stud-rails for the RC specimen consisted of seven headed studs welded to a bottom rail. Detailed information on the studs is provided in **Fig. 3-10**. The shaft of the stud had a yield stress of 635 MPa (92.1 ksi). The number of studs per rail and the rail length were selected to ensure that failure would occur within the shear-reinforced zone, versus outside the shear-reinforced zone (**Fig. 3-10**). Based on ACI 421.1R (1999), the first studs were located at 19 mm (= $0.27d \leq 0.40d$) away from the column faces, and studs were spaced at $0.5d$ = 35 mm on center based on an assumed shear crack angle of 45 degrees. Two stud-rails were placed perpendicular to each column face 187 mm apart. **Table 3-1** provides a comparison of the shear demand and capacity for both the potential slab critical sections.

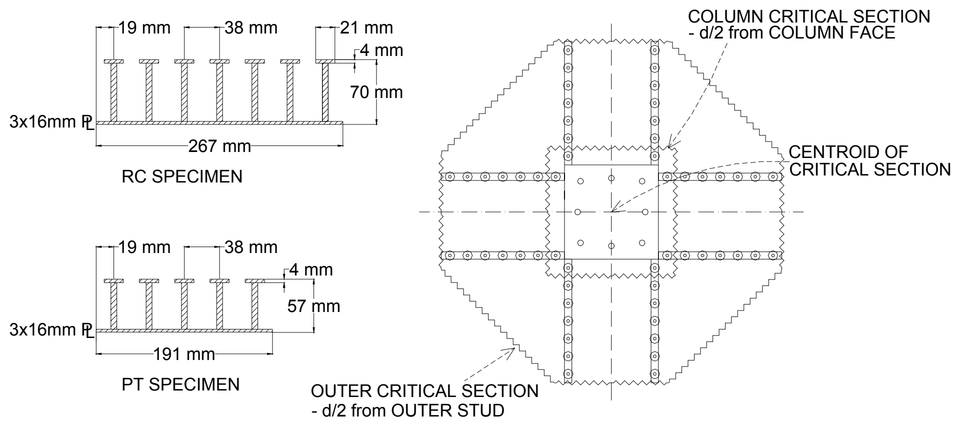


Fig. 3-10 Stud-rails and critical sections at connection regions

Design compressive concrete strength was 27.6 MPa (4,000 psi) with a maximum aggregate size was 9.5 mm (3/8 in.). Normal weight concrete mixed with Type II cement was used. Two placements were required to construct the RC specimen. Concrete for the column footings, first-story columns, and first-story slab was placed on February 15, 2002. Concrete for the second-story columns and second-story slab was placed on March 11, 2002. Measured mean compressive strength of the concrete used in the first and second concrete placements were 3,870 psi (26.7 Mpa) and 2,820 psi (19.4 MPa), respectively, based on core sample tests conducted soon after the lateral load testing in accordance with ASTM C-42 (**Table 3-3**). After finishing concrete surfaces, membrane-forming curing compounds were sprayed uniformly on top surfaces of slabs to prevent moisture loss. First and second-story wood forms were removed at the same time, hence the first and second-story concrete were exposed to air after 41 days and 16 days, respectively.

Table 3-3. Concrete compressive strength results of core samples

Sample I.D.	Sample Location	Recorded Compressive Stress [MPa]	Length To Depth	Correction Factor (ASTM C-42)	Corrected Compressive Stress [MPa]
RC-1-1	1st Floor Slab	27.87	1.38	0.95	26.48
RC-1-2	1st Floor Slab	26.40	1.43	0.95	25.10
RC-1-3	1st Floor Slab	29.94	1.46	0.95	28.48
					Average strength : 26.68
RC-2-1	2nd Floor Slab	19.06	1.43	0.95	18.13
RC-2-2	2nd Floor Slab	19.80	1.41	0.95	18.82
RC-2-3	2nd Floor Slab	22.49	1.40	0.95	21.37
RC-2-4	2nd Floor Slab	23.36	1.48	0.96	22.41
RC-2-5	2nd Floor Slab	17.22	1.46	0.95	16.34
RC-2-6	2nd Floor Slab	21.55	1.45	0.95	20.48
					Average strength : 19.60
PT-1-1	1st Floor Slab	23.36	1.32	0.94	22.00
PT-1-2	1st Floor Slab	18.31	1.29	0.93	17.03
PT-1-3	1st Floor Slab	18.55	1.31	0.94	17.44
					Average strength : 18.82
PT-2-1	2nd Floor Slab	24.33	1.29	0.93	22.62
PT-2-2	2nd Floor Slab	21.92	1.26	0.93	20.41
PT-2-3	2nd Floor Slab	19.99	1.30	0.94	18.82
					Average strength : 20.62

Concrete compression tests of 152 mm × 305 mm (6 in. × 12 in.) cylinders were conducted to determine the concrete strengths during the curing period. **Table 3-4** shows the compressive strengths of the first and second placements. For the first batch, the 28-

day strength is 202 MPa (2,933 psi), which is less than the target strength of 27.6 MPa (4,000 psi). The concrete cylinders set aside for evaluation on the day that the specimens were tested on the shake table were misplaced by the commercial testing laboratory. One remaining concrete cylinder of the first batch was tested to obtain stress-strain relations at UCLA structural laboratory four months after the completion of testing of the PT specimen. A peak stress of 26 MPa (3,773 psi) and strain at peak stress of 0.0024 were obtained. The secant modulus of elasticity to $0.45f'_c$ obtained from the stress-strain relation was approximately 19,305 MPa (2,800 ksi) (**Fig. 3-11**).

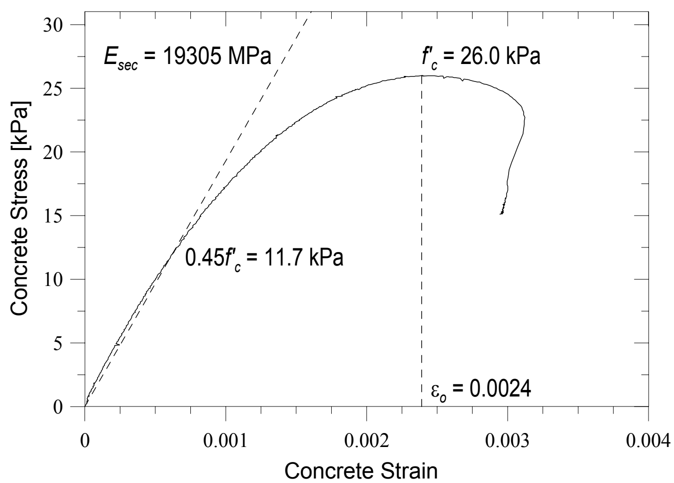


Fig. 3-11 Stress-strain relations of first batch concrete cylinder

Material properties obtained for the deformed reinforcing bars, stud-rail shafts, and seven-wire strands are shown in **Table 3-5,** and the associated stress-strain relations are depicted in **Appendix B**. Tensile testing was conducted in accordance with ASTM A370 and Annex A7 of ASTM A370. For stress-strain relations not displaying a distinct yield

point, a strain offset of 0.2 percent was used to define the yield stress. **Table 3-5** summarizes material properties for the concrete and the steel (e.g. the average compressive concrete strength) used for the analyses presented in future chapters.

Table 3-4. Concrete compressive strength results of cylinders

Sample ID	Age	Dimension (mm)	Fracture Type*	Compressive Stress [MPa]
1 batch-1	7	152.4 × 304.8	C	14.07
1 batch-2	7	152.4 × 304.8	D	14.20
1 batch-3	14	152.4 × 304.8	D	18.27
1 batch-4	14	152.4 × 304.8	C	17.51
1 batch-5	28	152.4 × 304.8	D	19.93
1 batch-6	28	152.4 × 304.8	D	20.00
1 batch-7	28	152.4 × 304.8	C	20.75
1 bacth-8	35	152.4 × 304.8	D	21.51
1 bacth-9	35	152.4 × 304.8	D	21.37
2 batch-1	7	152.4 × 304.8	C	18.62
2 batch-2	7	152.4 × 304.8	C	17.93
2 batch-3	14	152.4 × 304.8	C	23.51
2 batch-4	14	152.4 × 304.8	C	24.20
2 batch-5	28	152.4 × 304.8	C	29.44
2 batch-6	28	152.4 × 304.8	C	29.37
2 batch-7	28	152.4 × 304.8	C	29.17

* Fracture Types: A=cone, B=cone and split, C=cone and shear, D=shear, E=columnar

Table 3-5. Material properties of steel bars and seven-wire strands

Sample I.D.	Diameter [mm]	Area [mm^2]	Yield Strength [MPa]	Tensile Strength [MPa]
#2 – 1	4.09	13.16	$450.9^{*}/355.1^{**}$	662.6
#2 – 2	4.09	13.16	$493.7^{*}/405.4^{**}$	689.5
#2 – 3	4.06	12.97	$499.2^{*}/359.9^{**}$	699.8
Average yield strength: $481.3^{*}/373.5^{**}$ [MPa]				
#3 – 1	6.35	31.68	$458.5^{*}/417.1^{**}$	659.8
#3 – 2	6.30	31.16	$465.4^{*}/392.3^{**}$	650.9
#3 – 3	6.30	31.16	$449.5^{*}/392.3^{**}$	645.3
Average yield strength: $481.3^{*}/373.5^{**}$ [MPa]				
#4 – 1	9.04	64.19	490.9	697.1
#4 – 2	9.07	64.58	475.0	694.3
#4 – 2	9.04	64.19	507.4	701.2
Average yield strength: 491.1 [MPa]				
Studrails – 1	3.99	12.52	630.9	732.2
Studrails – 2	4.01	12.65	632.9	746.0
Stdurails – 3	4.09	13.16	640.5	750.1
Average yield strength: 634.8 [MPa]				
Strands – 1	7.95	37.23	1646.5	1840.2
Strands - 2	7.95	37.75	1603.0	1827.1
Average yield strength: 1624.8 [MPa]				

* Yield Strength @ 0.2% Offset, ** Strength at 0.002 strains.

3.2 PT Specimen

The configuration of the PT specimen was similar to that used for the RC specimen (**Fig. 3-12**), except a slab thickness of 76 mm (3 in.) and bay widths of 2.84 m (9 ft. 4 in.) in the direction of testing were selected (**Fig. 3-13(a)**). The resulting span-to-depth ratio of 37.3 is slightly less than typical values used for non-participating PT slab-column frame construction on the west coast of the US (40 to 45). The specimens were constructed outside the laboratory; therefore, the width of the laboratory door (5.8 m; 19 ft.) limited the span length to (2.64 m; 8 ft. 8 in.) between columns perpendicular to the direction of loading, with overhanging "one-half" span lengths shortened to 1.17 m (3 ft. 10 in.) on each side (**Fig. 3-13(b)** and **3-14**). Column geometry and reinforcement of the PT specimen were the same as those of the RC specimen, resulting in a column depth-to-span length ratio of $c_1/l_1 = 0.07$, which is fairly typical of PT flat plate construction. The use of identical footing and column size allowed the reuse of many items. Lead weights were affixed to the 76 mm (3.0 in.) floor slabs to provide 3.6 kPa (76 psf) of gravity load, to result in a total slab self-weight of 5.5 kPa (114 psf), which corresponds closely to the self-weight of the prototype 229 mm (9 in.) thick post-tensioned slab. The resulting gravity stress ratios on the interior and exterior connections are approximately $0.33V_c$ ($0.39\phi V_c$) and $0.25V_c$ ($0.30\phi V_c$), which are less than the limit of $0.4\phi V_c$ for intermediate slab-column frames of ACI 318-02 21.12, but slightly low for typical non-participating slab-column frames used in the US, which are typically in the range of $0.35V_c$ to $0.45V_c$, and may be as high as $0.60V_c$.

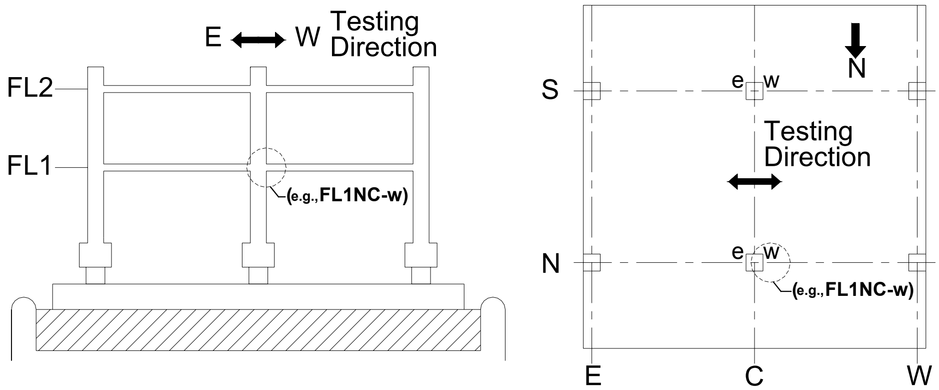


Fig. 3-12 Test specimen on shake table

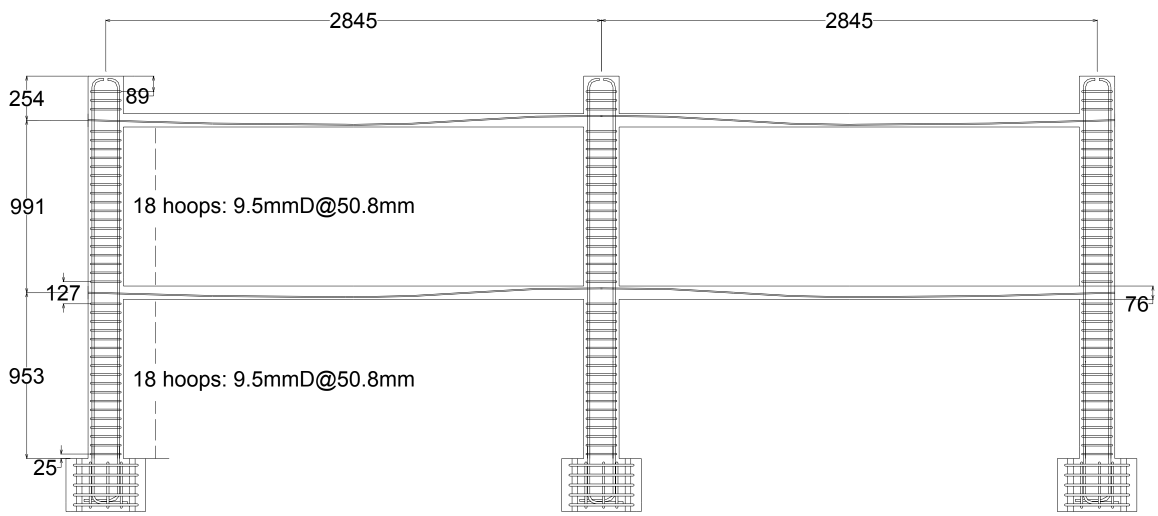
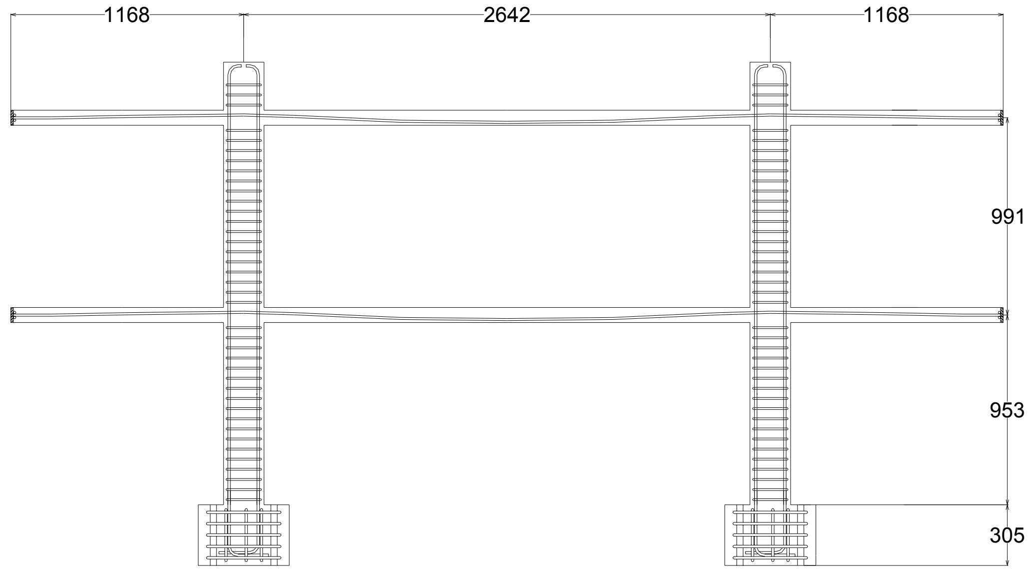


Fig. 3-13(a) Elevation view of PT specimen (Frame N, S)



FRAME E, FRAME C, FRAME W (See Fig. 3-12)

Fig. 3-13(b) Elevation view of PT specimen (Frame E, C, W)

Tendons were placed according to ACI 318-02 ("Building", 2002) and ACI 423.3R (1996) provisions and recommendations, respectively. Tendon arrangements are shown in **Fig. 3-14** and **3-15(a)**. Six banded tendons, with two passing through the column cage, were used along each column line in the E-W direction (direction of testing), and seven (two passing through the column cage) distributed tendons between each column line were used in the N-S direction (**Fig. 3-15(b)** and **3-15(b)**). An effective tension force of 44 kN (9.9 kips) per strand was applied based on flexural strength requirements, resulting in an average compressive stress of 1.39 MPa (202 psi) in the banded (testing) direction (E-W) and 1.37 MPa (199 psi) in the other direction (N-S). Given the effective tendon force, the post-tensioned slab balanced a gravity load of 4.3 kPa (90 psf), or 79% of dead

load (including lead-weights, which are discussed in more detail in Chapter 4, Section 4.1.).

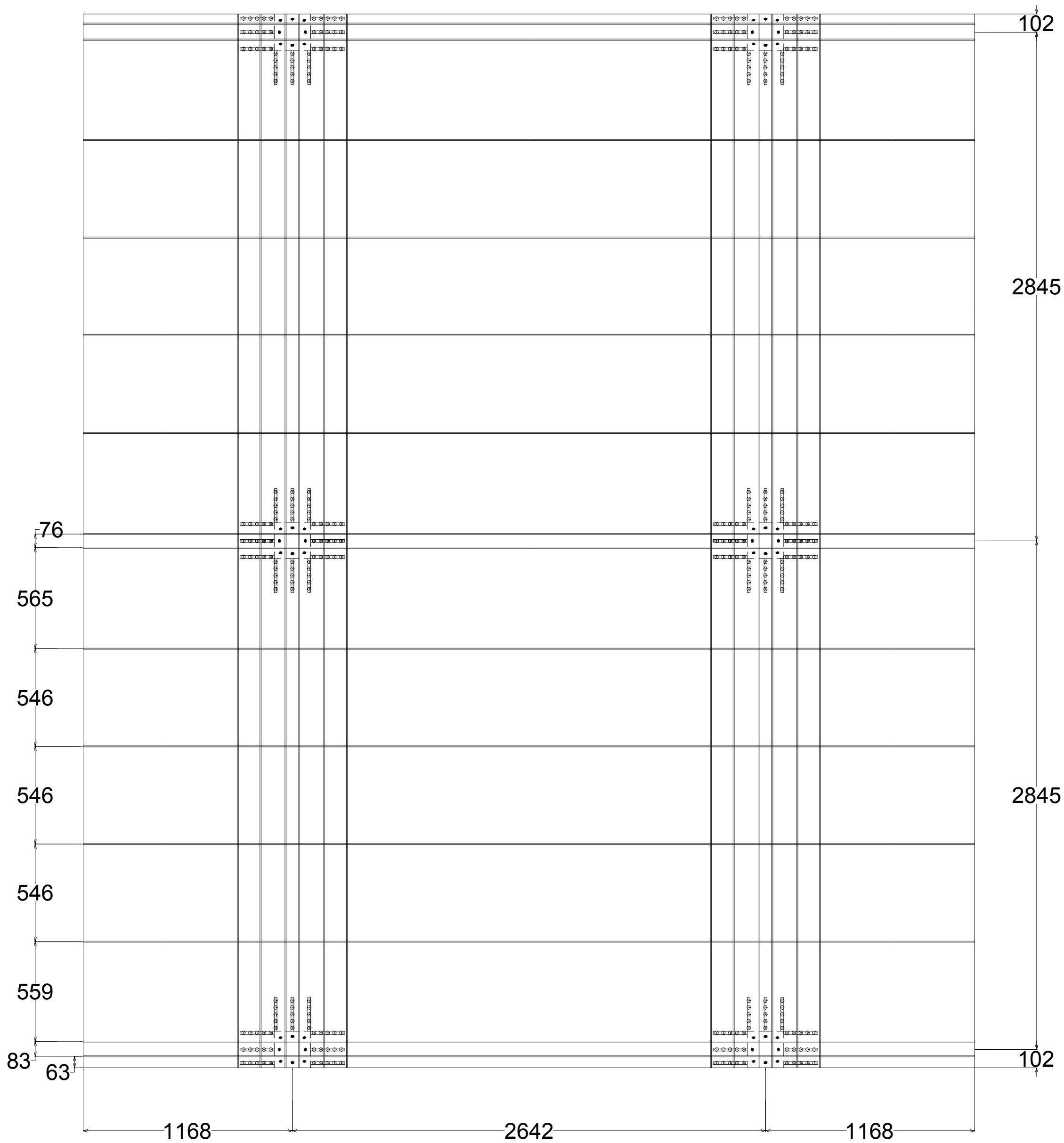


Fig. 3-14 Tendon arrangement of PT specimen

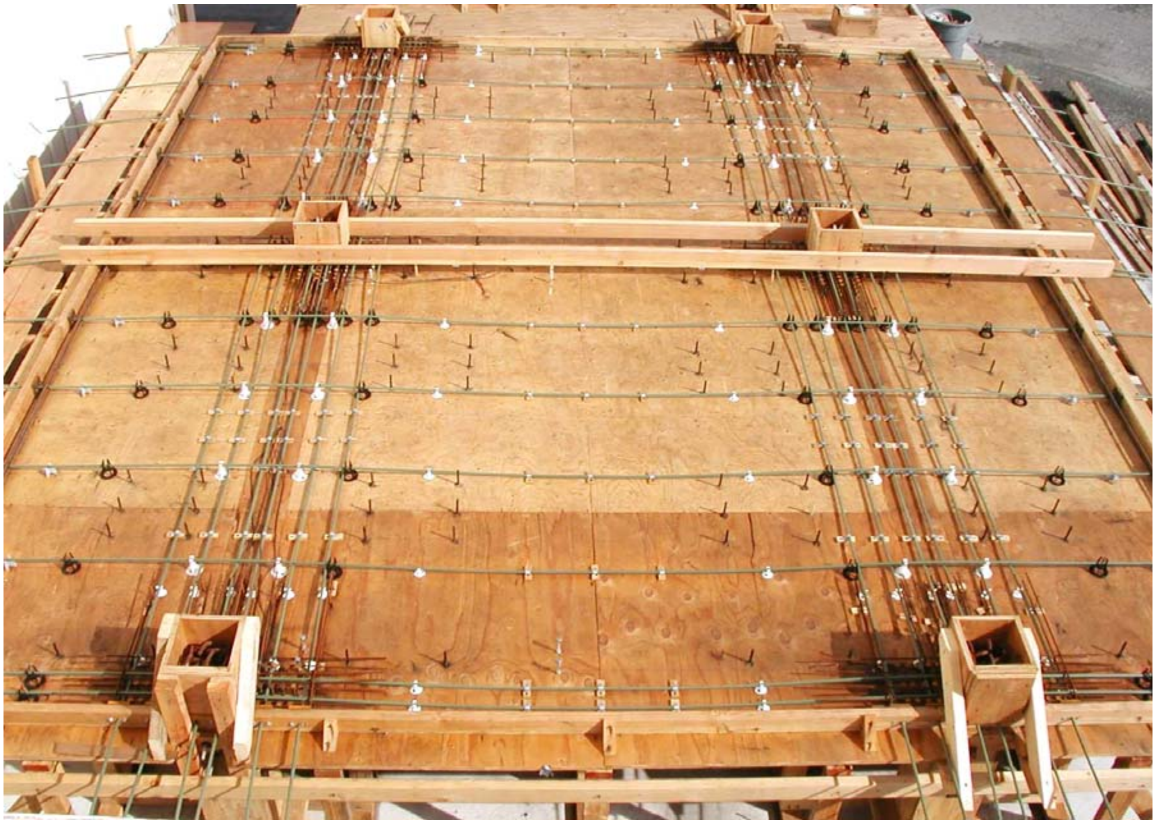


Fig. 3-15(a) Overview of tendon arrangement (PT specimen)

Fig. 3-15(b) Interior connection (PT)

Fig. 3-15(c) Edge connection (PT)

Anchor plates (76.2 × 76.2 × 12.7 mm; 3 × 3 × 1/2 in.) with 16 mm (5/8 in.) diameter holes were used to provide bearing against the concrete for the tendons (**Fig. 3-16**). In order to transfer the post-tensioning force to the concrete, reusable barrel anchors were installed at jacking and dead ends, which were alternated.

Fig. 3-16(a) Anchor plate and edge tension bar

Fig. 3-16(b) Anchors

Bonded top reinforcement was provided at connection regions as required by ACI 318-02 S18.9.3.3 (**Fig. 3-17**). For interior connections, 6 − # 2 (d_b = 6.35 mm; 0.25 in.) top and bottom bars with a yield stress of 481 MPa (69.8 ksi) were provided in both directions (**Fig. 3-15(b)** and **3-18(a)**). For edge connections, 8 − # 2 (d_b = 6.35 mm; 0.25 in.) hairpin bars, and 4 − # 2 (d_b = 6.35 mm; 0.25 in.) top and bottom bars were placed in the E-W direction (direction of testing) and N-S direction, respectively (**Fig. 3-15(c)** and **3-18(b)**). Although not required, an equal quantity of bottom reinforcement was provided to control cracking along the bottom of the slab anticipated due to applied dynamic (cyclic) load history as well as to improve the hysteretic energy dissipation capacity of the system.

Bonded bars provided to control cracking at edge connections also provided reinforcement to the anchorage zones. In addition, edge tension reinforcement was provided at the anchorage zones to resist bursting pressures (**Fig. 3-17** and **3-18(b)**). Unbalanced moments at the connections were computed assuming a yield line would form across the entire width of the slab. Given the quantity of post-tensioning tendons and bonded reinforcement within the column strip, as well as the total effective strand forces in the direction of loading, shear reinforcement at the slab-column connections was required to avoid punching failures for the design level forces.

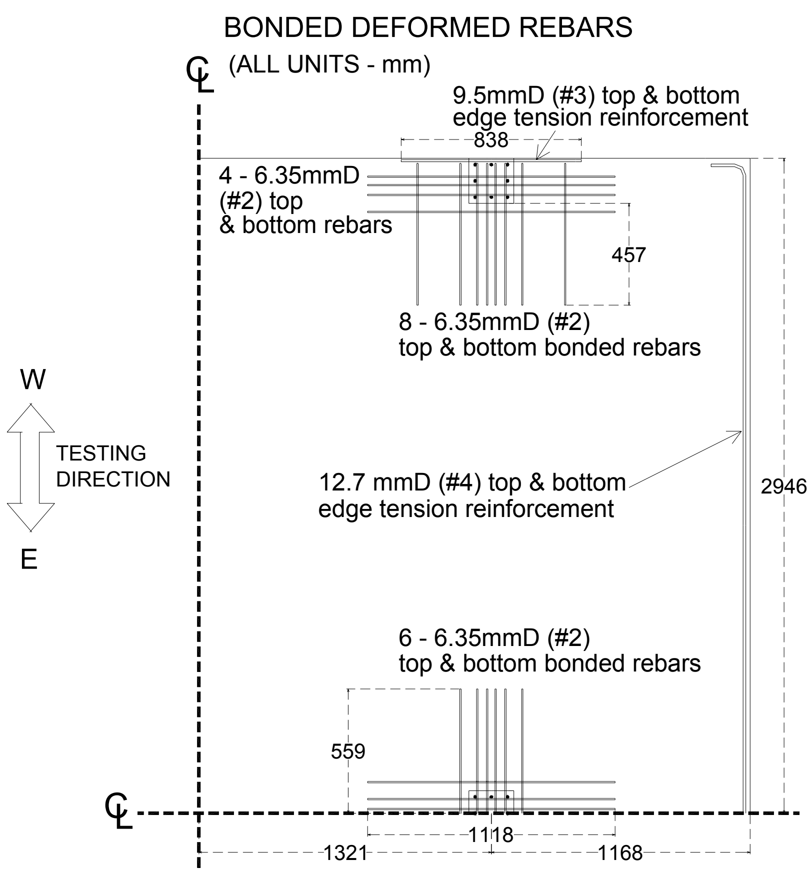


Fig. 3-17 Details of bonded reinforcement (PT specimen)

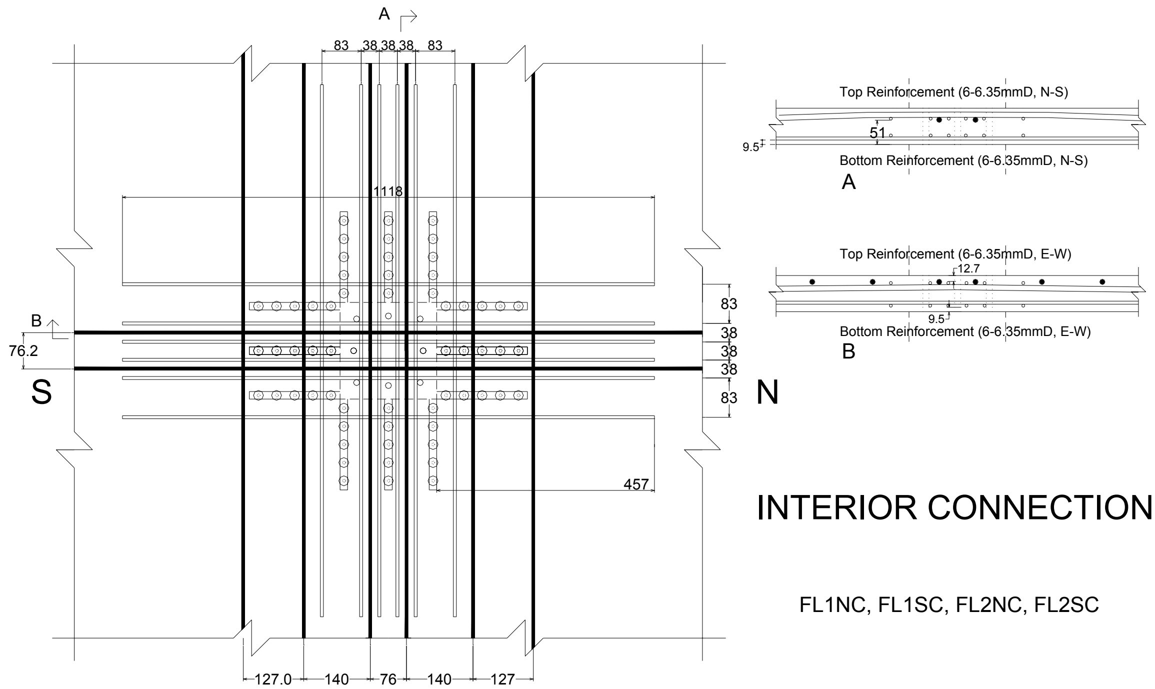


Fig. 3-18(a) Details of interior connection (PT specimen)

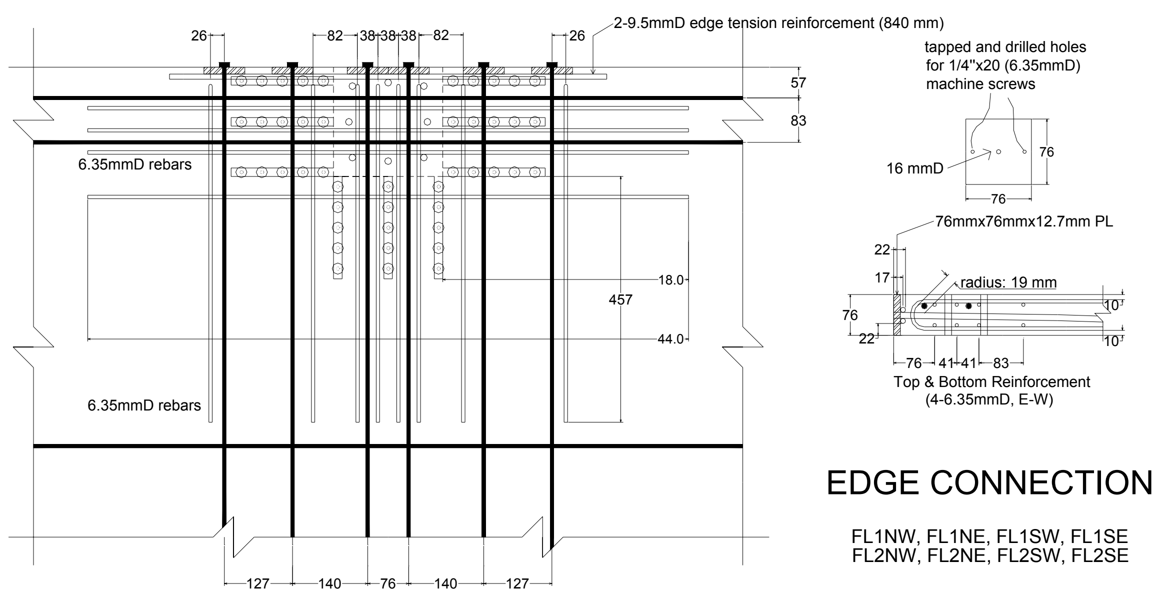


Fig. 3-18(b) Details of edge connection (PT specimen)

Shear reinforcement for the PT specimen consisted of stud-rails (**Fig. 3-18(a)** and **3-18(b)**) satisfying requirements of ACI 318-02 ("Building", 2002) and ACI Committee Report 421.1R (1999). The size and spacing of the stud-rails were selected to require flexural yielding of bonded bars prior to punching failure on a critical section *d/2* from the column face. The geometry of the studs used in the PT and RC specimens were identical, except for the length of the stud shaft due to the variation in slab depth (76 mm; 3in. for PT, 89 mm; 3.5 in. for RC). Five studs per rail were selected to produce a stud-rail that extended *2.5h* (note that the minimum length of *4h* will be required in the ACI 318-05 provisions) from the column face, to ensure that the punching failure would occur within the shear reinforced region (at the slab critical section *d/2* from the column face). Three stud-rails were placed perpendicular to each column face.

The PT specimen was constructed at the same time as RC specimen. Sufficient platforms and shores were provided to support the concrete self-weight of slabs (1.8 kPa; 37.5 psf). Holes were drilled in slab edge forms to allow for the tendons to be placed (**Fig. 3-16(a)**). Anchor plates with two-drilled and tapped holes for 1/4"$\phi \times$ 20 (6.35 mm diameter) machine threaded fasteners were attached against the inside edge of the forms, ensuring the plates are perpendicular to the forms (**Fig. 3-16(a)** and **3-18(b)**). Rebar chairs were purchased or fabricated to ensure tendons were installed as specified on the drawings (**Fig. 3-19**) within a tolerance of 3.2 mm (1/8 in.). Accurate installation of tendons ensured that wobble friction caused by unintended curvature was minimized. Minimum clear cover was 9.5 mm over both the top and bottom of tendons or bonded reinforcement

(#2, d_b = 6.35 mm), except at interior connections where top clear cover of 7.9 mm and 12.7 mm were designated for tendons and bonded (deformed) reinforcement, respectively, due to space limitations (**Fig. 3-19**).

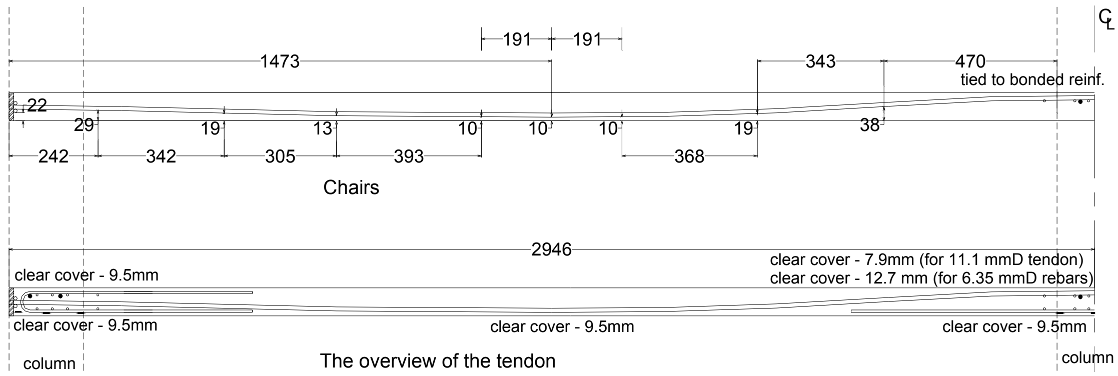


Fig. 3-19(a) Tendon profiles (E-W direction, See Fig. 3-12)

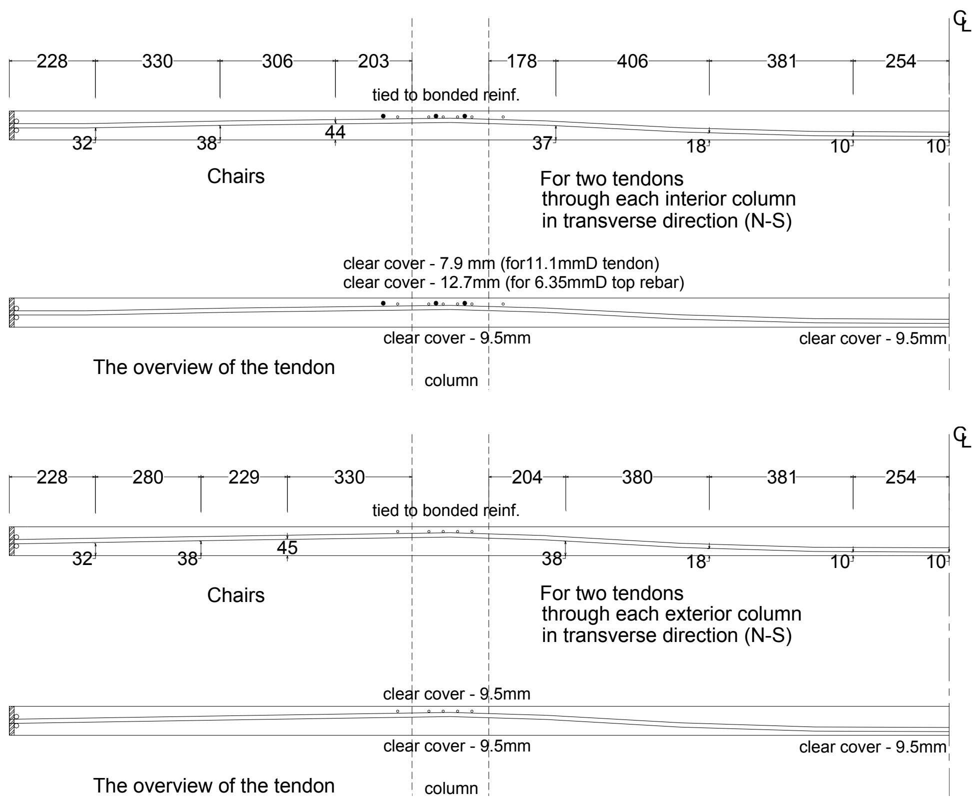


Fig. 3-19(b) Tendon profiles (N-W direction, See Fig. 3-12)

The PT specimen was cast in place using the same concrete as was used for the RC specimen. After casting, concrete slabs were sprayed of curing compounds. Mean compressive strengths of 18.3 MPa (2,730 psi) and 20.6 MPa (2,990 psi) were obtained for the first-story and second-story slabs of PT specimen, respectively, based on core sample tests done in accordance with ASTM C-42 (**Table 3-3**). All wood forms for the PT specimen were removed at the same time as the removal of forms for the RC specimen, except for the slab edge forms, which were detached just prior to post-tensioning.

Before shores were removed, all tendons were partially tensioned to an average of 19.6 kN (4.4 kips) per tendon to balance the slab self-weight. The jack pressure was calibrated by using donut-shaped load cell, which measured the post-tensioning force directly (**Fig. 3-20**). Partial post-tensioning work for the first and second stories was conducted 5 and 8 days after each concrete placement, respectively. The post-tensioning process first involved removing the slab edge forms and removing all exposed plastic sheathings on the 7.94 mm (5/16 in.) strands beyond the bearing plates. Protruding strands were cut-off 43 cm (17 in.) from the slab edge prior to moving the specimen into the laboratory. The 43 cm (17 in.) length was sufficient to allow the post-tensioning jack to grip the tendons for final post tensioning, which was done during the installation of the lead weights. The installation of the lead weights and the final post-tensioning are discussed in Chapter 4, Section 4.1.

Fig. 3-20 Donut-shaped load cells

3.3 Summary

Two, approximately one-third scale replicas of a reinforced-concrete (RC) flat-plate system and a post-tensioned (PT) flat-plate system, were constructed to be tested at the UC Berkeley Richmond Field Station. Each specimen consisted of a 2-by-2 bay slab-column frame, two stories high. For the RC specimen, a span length of 2.06 m and a slab thickness of 89 mm were used, whereas 2.84 m spans and a 76.2 mm thick slab were used for the PT specimen. The resulting slab span-to-depth ratios of 23.1 (RC) and 37.3 (PT) are within typical values used for RC (15 to 40) and PT (35 to 45) construction. Columns cross sections were 203 mm x 203 mm reinforced with 8 – 12.7 mm diameter longitudinal bars with a nominal yield stress of 414 MPa. Design concrete compressive strength was 27.6 MPa. Slab reinforcement for the RC and PT specimens consisted of 9.5

mm and 6.35 mm diameter bars, respectively, whereas post-tensioning consisted of 7.94 mm nominal diameter, seven wire strand with an ultimate strength of 1,725 MPa. The gravity shear ratios for the interior connections of the RC and PT specimens were 0.25 and 0.33, respectively, for the design concrete strength of f'_c = 27.6 MPa. The connection regions for both specimens were designed such that punching failures were expected to occur prior to yielding of slab flexural (column strip) reinforcement in the absence of slab shear reinforcement. With the addition of slab shear reinforcement, yielding of slab reinforcement within the column strip was anticipated; however, punching failures were expected to occur prior to yielding of slab reinforcement across the full width of the slab.